THE MECHANICS OF DEEP-SEATED LANDSLIDES

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ABSTRACT

Results are presented of a sequence of laboratory tests undertaken to elucidate the behaviour of deep-seated landslides. In deep-seated failures deformation has been reported at depths of up to 250 m. In the movement zone, owing to the weight of the overburden and the surrounding stress environment, conventional soil mechanics cannot be used to explain effectively associations between the landslip activity and the deformation mechanisms operating within the moving mass. A series of experiments has been undertaken on London Clay using a high pressure, servohydraulically controlled triaxial deformation system, to replicate the stresses acting at the base of a large, deep-seated landslide. A number of tests were undertaken, the most significant focusing on the transition between ductile and brittle behaviour. Although sediments usually behave in a brittle manner at low effective stresses (common to many geomorphological studies) and in a ductile manner at high effective stresses, the results presented here identify for the first time in mudrocks a transitional phase of behaviour in which creep-like movement will manifest itself at the base of a deep-seated landslide as the growth of microcracks. The microcracks may eventually coalesce to form a shear surface, a consequence of which is likely to be sudden failure. The results thus have important implications in the understanding of movement mechanisms in large, deep-seated failures, rates of displacement and how they may change through time. © 1997 by John Wiley & Sons, Ltd.

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INTRODUCTION

Many terrestrial mass movements are shallow-seated failures, which move across bounding shear surfaces. Many landslides result in the displacement of discrete blocks of material such as translational slides and rotational slides (Hansen, 1984; Allison, 1992). Others, such as mudslides (Brunsden, 1984) and debris flows (Davis, 1992), involve masses of softened material, frequently with a high moisture content, where material is transported downslope at varying speeds depending on external factors controlling shear stress and internal factors affecting shear strength (Terzaghi, 1950). In most cases, terrestrial mass movements can be examined by utilizing established methods in soil mechanics (Lambe and Whitman, 1979; Anderson and Richards, 1987; Muir-Wood, 1990) and rock mechanics (Wittke, 1991; Hoek, 1992). Recent examples include a study of the effects of weathering on the stress–strain behaviour of Hong Kong saprolitic soils (Au, 1993), the modelling of rockfall activity in the Canadian Rocky Mountains (Cruden and Hu, 1993) and examining the consequences of reduced shear strength for Tertiary calcareous mudrock earthflow activity on the North Island of New Zealand (Trotter, 1993).

In marked contrast to shallow-seated failures (Chandler, 1986) is the relatively rare phenomenon of sudden, deep-seated landslides, in which mechanical deformation occurs at considerable depths. Deep-seated landslides have been identified in many parts of the world; however, in the context of this research many previous mass movement studies such as the Minor Creek landslide in northwest California (Iverson, 1986), which have been described as deep-seated failures with shear zones typically at a depth of 5 m, are regarded as shallow failures. Examples of deep-seated slides in the context of this study include the Vaiont landslide in

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northern Italy (Skempton, 1966; Radbruch-Hall, 1978; Voight and Faust, 1982, 1992), and instability in the Coledale area of the Illawara Escarpment, New South Wales, Australia (Walker *et al*., 1987).

An important but nevertheless little examined and poorly understood aspect of deep-seated failures is their geomechanical behaviour, particularly at depths in excess of 100 m. The mechanical characteristics and deformation of soft sediments at such depths differ significantly from those in the near-surface environment, primarily due to the weight of the overburden and stress environment. Despite this, there is virtually no evidence that geomorphologists have considered the implications of the stress–strain regime relevant to large failures. Indeed, published material tends to concentrate almost exclusively on failure magnitude (e.g. Ohmori, 1992) and rates of displacement (e.g. Azzoni *et al*., 1992). Examples can be found of studies where the geomechanics of failures are examined by recourse to soil mechanics theory relevant only to the near-surface environment (e.g. Choubey and Rawat, 1990).

Movement patterns in deep-seated landslides have been extensively documented (e.g. Radbruch-Hall, 1978; Pasuto and Soldati, 1990), with two distinct patterns being evident. First, there may be long-term displacement at low strain rates, frequently termed ‘creep’. Creep may vary in rate on a seasonal basis but movement will rarely cease altogether. Second, there are cases of short-term movement at very high rates of displacement, representing a sudden failure. The relationship between creep and catastrophic displacement is complex, but may be divided into a number of basic patterns. Some deep-seated landslides creep for long periods under normal gravitational forces. In such cases rates may be relatively constant. Minor fluctuations may be the result of small changes in the water-table, which in turn alters the effective normal stress. Creep may occur in increments, possibly controlled by pore pressure, or may be triggered by seismic events. An example of apparent tectonic influence is the behaviour of the Point Firmin landslide, California (Miller, 1931), in which rates of creep were oberved to increase from 3 cm per week to 15 cm per week after a small earthquake.

Deep-seated landslides may undergo short periods of creep culminating in sudden failure. The short period of creep will commonly show an increase in displacement rate immediately prior to the rapid strain event. An example of such movement is the Turtle Mountain landslide near Frank, Alberta in 1903 (Mudge, 1965). Finally, a few deep-seated landslides show long-term, steady-state creep behaviour that changes to sudden failure. Examples of such activity are the 1963 Vaiont reservoir disaster in northern Italy (Radbruch-Hall, 1978) and the Goldau landslide in Switzerland (Ter-Stepanian, 1966). At Vaiont, the construction of a reservoir initiated creep in 2×10^8 million m^3 of interbedded limestone and shale which continued for approximately three years before a change in mechanism to sudden failure. There is evidence to suggest that during the creep phase the rate of displacement of the Vaiont landslide was closely correlated with the height of the water-table.

STRESS–STRAIN RELATIONSHIPS IN LANDSLIDE SYSTEMS

The deformation of geological materials is commonly considered to occur in two regimes, one brittle and the other ductile (Engelder, 1993). Brittle deformation can be illustrated in a standard stress–strain curve (Figure 1). As a load or stress is applied to a material it initially undergoes a short phase of elastic (recoverable) strain. This represents a situation where the bonds which result from grain–grain adhesion between the particles (Johnson, 1985) within the material are being loaded but are not breaking. Eventually the load will become sufficiently large that the weakest or most intensely stressed bonds will begin to break. At this point the material is undergoing a combination of elastic and plastic deformation, characterized by a steady decrease in the gradient of the stress–strain curve. Eventually the material will reach a point at which so many bonds have been broken that it is unable to bear the applied load or stress. At this point a shear surface will develop as a consequence of strain weakening, and strength will decrease. Thereafter deformation in the material will occur primarily as displacement across the shear surface, with the friction across it determining the material strength. Once the shear surface is fully developed, strength will settle to a residual value. Brittle failure commonly occurs in bonded or cemented materials at relatively low confining pressures. During deformation at confining pressures in the range 1–250 kPa, most engineering soils will display brittle behaviour. Mudrocks may also display brittle behaviour at confining pressures in the range 0–2 MPa. Harder geological materials will deform in the brittle regime up to greater confining pressures.

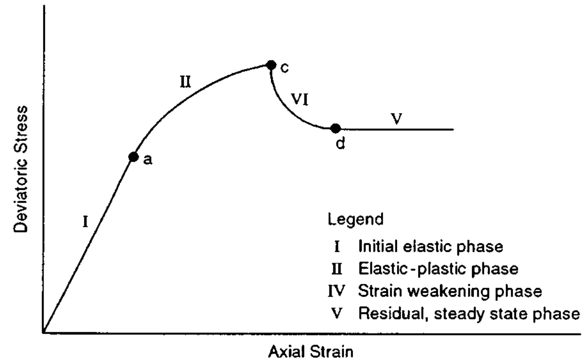

Figure 1. An idealized stress–strain curve illustrating the characteristics of brittle deformation

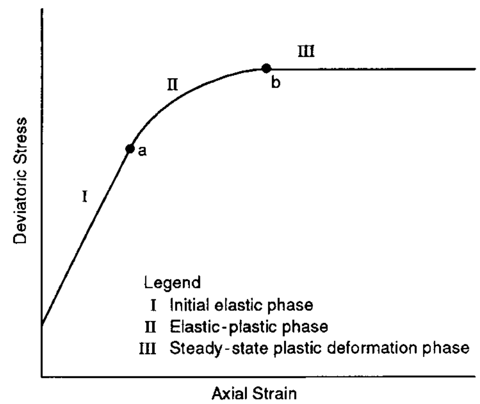

Figure 2. An idealized stress–strain curve illustrating the characteristics of ductile deformation

Ductile behaviour is displayed by most geological materials at high effective stresses and may also be shown by materials with little or no interparticle bonding, such as weathered clays (Fan, 1994; Fan *et al.*, 1994) (Figure 2). In the ductile case an initial phase of elastic deformation is followed by a phase of elastic/plastic deformation resulting from inter-particle breakage, similar to that shown by brittle materials. What is critical here is that strain is not able to localize to form a single shear plane owing to the high confining pressure. In effect, the material gradually becomes destructured as all interparticle bonds are broken. The material is not able to sustain increases in load but at the same time cannot strain-weaken, resulting in a phase of purely plastic, or cataclastic, deformation at constant stress. The mechanism for the deformation is thus an internal restructuring of the material. In theory, the material is able to continue to deform in this condition to infinite strain, although in reality a gradual decrease in strength may occur as particle reorientation proceeds. In the natural environment most geological materials can display both forms of deformation depending on their confining pressure (Cristescu, 1989). At low confining pressures, such as those representative of shallow slope failures, rocks will tend to display brittle deformation, superseded by ductile deformation if confining pressures reach a sufficiently high value. A good example is chalk (Figure 3), where both types of strain response can be identified (Leddra *et al.*, 1993). At low confining pressures chalk behaves in a brittle manner, but at higher values the material becomes ductile owing to the destructuring of the mineral matrix. Thus a transition occurs depending on the physical characteristics of the material and the local stress environment.

Of particular significance to this research is the fact that the brittle and ductile deformation mechanisms can be directly related to the movement patterns of deep-seated landslides. Sudden failure in deep-seated landslides occurs because the materials in the vicinity of the basal shear surface have undergone brittle failure, evidenced by the formation of the shear surfaces. The development of the shear surface under brittle failure conditions

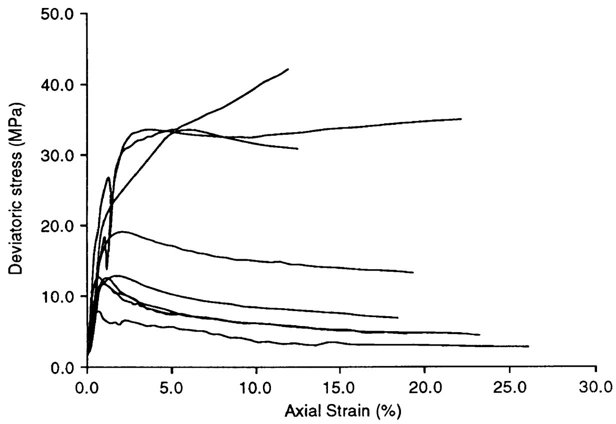

Figure 3. A group of stress–strain curves for chalk, illustrating ductile deformation at high confining pressures and brittle deformation at low confining pressures (after Leddra *et al*., 1993)

leads to a weakening of the material at the base of the landslide. Since the stress system is effectively static, the decrease in strength must be balanced by an increase in the rate of strain. As the material undergoes a rapid loss of strength at brittle failure, there will be an increase in the rate of strain. Once movement has been initiated, the moving mass may be displaced by a large amount and slip will occur until a new stable state is reached. Strain-weakening, and hence increases in the rate of displacement, may be further promoted by polishing of the shear surface as clay particles become aligned and by frictional heating of pore fluids. Voight and Faust (1982, 1992) suggest that the high rate of movement, in the region of $25\,m\,s^{-1}$, attained by the Vaiont slide was the result of frictional heating of the pore fluid along the thin argillaceous bands that formed the shear surface.

Many large landslides undergo a short period of creep prior to sudden failure. Creep occurs as the material is steadily deformed during the elastic and elastic/plastic phases. Whilst deformation in the elastic phase will be at an approximately constant rate of strain, once yield is reached the material is effectively undergoing progressive destructuring as interparticle bonds are broken. Thus in a static stress system, the rate of strain may increase as the material undergoes progressive damage during the loading process. Failure of the landslide will occur once a critical strain is attained, that strain being determined by the amount of damage to the structure of the deforming mass. Thus the behaviour of landslides that undergo short phases of creep terminated by abrupt brittle failure is easily described by the behaviour of initially intact, brittle materials.

The behaviour of landslides that undergo long, very stable phases of creep, abruptly terminated by sudden, perhaps catastrophic failure, is more difficult to explain using conventional soil and rock mechanics. Long phases of creep suggest that the material is undergoing ductile deformation at a steady state. Minor changes in rates of displacement in this case are attributable to fluctuations in the water-table. Counter to this, however, is the fact that sudden failure suggests the material is deforming in the brittle regime. In other words, deformation appears simultaneously to be a combination of both the brittle and ductile regimes.

The paradox of simultaneous brittle and ductile conditions presents a problem in that it does not lie within conventional soil mechanics. An experimental programme was undertaken to study the phenomenon further and investigate the behaviour of argillaceous rocks, the most common rock type at the surface of the Earth, under simulated deep-seated landslide conditions. The experiments were designed to replicate the stress conditions at the base of a deep-seated landslide, allowing for the analysis of the mechanisms responsible for landslide movement.

EXPERIMENTAL PROCEDURE

A series of laboratory experiments was undertaken on undisturbed samples of London Clay. London Clay was selected because of extensive documentation on its properties (Table I). Although there are no reported examples of deep-seated failures in London Clay, documentary evidence does not suggest that their

Table I. The properties of London Clay

Moisture content	28–33%
Plastic limit	32
Liquid limit	67
Plasticity index	35
Preconsolidation pressure	1400 kPa
Angle of internal friction, ϕ'_p	25–29°
Effective cohesion, C'_p	108–252 kPa

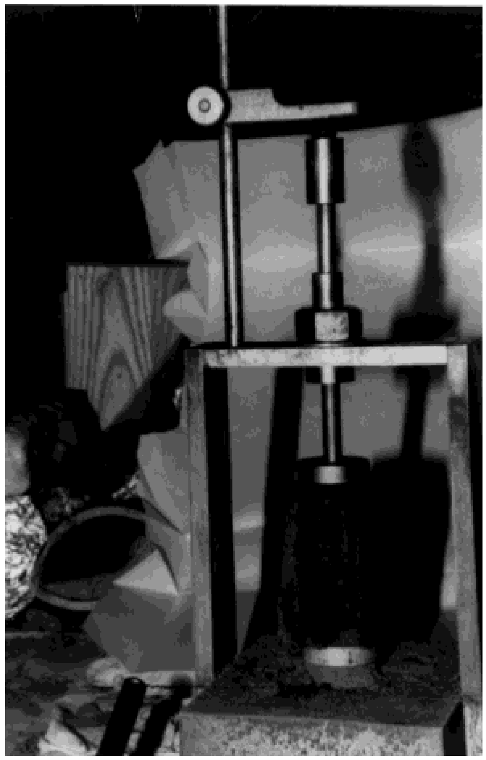

Figure 4. Sample preparation using the soils lathe

development is physically impossible, and other factors, such as extremely well documented material behaviour in the low-stress regime, make London Clay ideal for a study of this nature. London Clay is an over-consolidated, grey, Eocene mudrock, which outcrops extensively in southeast England and is prone to landslip activity, particularly at the coast (Hutchinson, 1979). Undisturbed cylindrical samples, 38 mm in diameter and 76 mm in length, were prepared using a soils lathe (Figure 4) to minimize disruption (BSI, 1992). Samples were tested in high pressure triaxial cells with a servohydraulic control system (Figure 5), allowing confining pressures and back pressures in the range 1–70 MPa to be applied. The experiments and equipment are described in detail elsewhere (Petley, 1994). The sample was placed in a protective rubber membrane within the triaxial cell (Figure 6).

To replicate the stresses acting at the base of a large, deep-seated landslide, each experiment comprised two distinct phases. In the first phase the sample was subjected to a condition simulating the weight of a theoretical representative overburden. In other words, the test material underwent a phase of compaction by incrementally applying pressure via the confining fluid surrounding the sample. The sample was allowed to consolidate and pore fluid was expelled into a volume gauge. Owing to the low permeability of London Clay, the compaction phase lasted an average of six weeks for each test. Once the sample had been stabilized at an effective stress representative of that at the base of a deep-seated landslide, a shear stress was applied. The sample was loaded axially at a constant displacement via a ram connected to a 250 kN load frame. During the loading phase, the state of stress and strain was accurately measured, allowing an analysis of sample deformation.

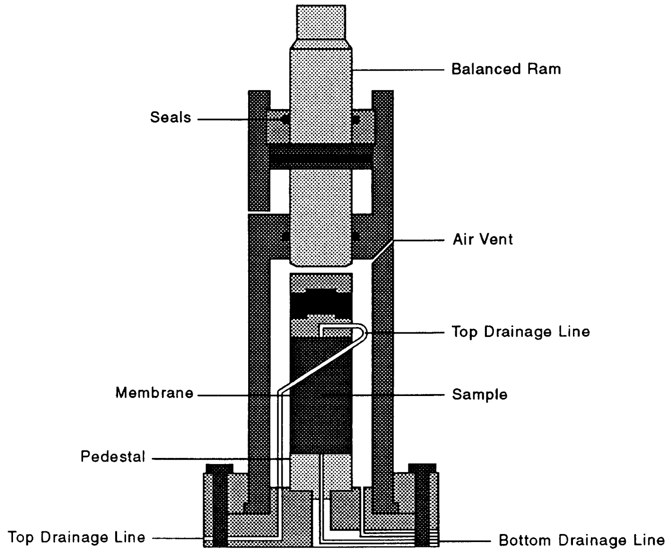

Figure 5. Main components of the triaxial apparatus

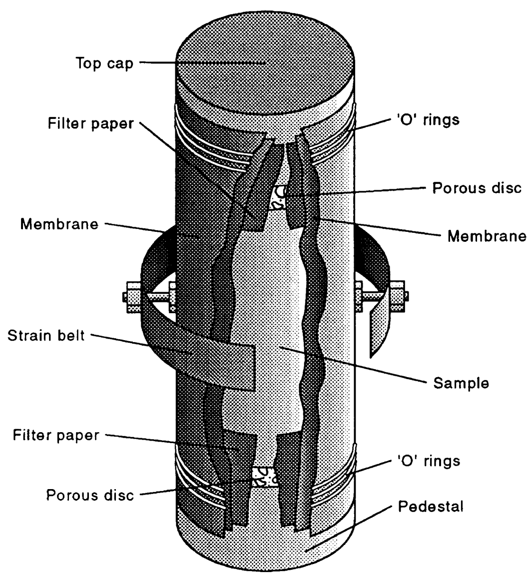

Figure 6. Sample configuration used for triaxial tests

Care must be taken in the interpretation of the results. It is not possible to directly compare behaviour of materials in a triaxial cell to that in natural systems because subtly different loading conditions apply. In the triaxial cell, loading is achieved by driving a ram onto the sample, effectively setting a constant rate of strain at the top of the sample and allowing the stress to develop as a result. In natural systems the rate of strain is controlled by the stress and pore fluid conditions. Nevertheless, this is a constraint in any triaxial test, be it at high or low pressure, where the results are used in landslide stability analysis and it is therefore not unique to this test programme. The deformation mechanisms that are acting within the landslide on the one hand and the triaxial cell on the other are analogous, allowing the comparison of (for example) ductile and creep

Table II. Characteristics of experiments undertaken in the study

Sample	*Po′* (MPa)	Maximum strain (%)	Pore pressure (MPa)
Luuncud 1	2·2	9	1·00
Luuncud 2	6·4	22	0·96
Luuncud 3	19·0	20	0·91
Luuncud 4	30·3	21	1·00
Lonpart 1	6·3	21	0·94
Lonpart 2	6·2	15	0·97
Lonpart 3	6·0	7·6	0·96
Lonpart 4	6·2	2·5	0·88
Lonpart 5	6·2	10	0·87

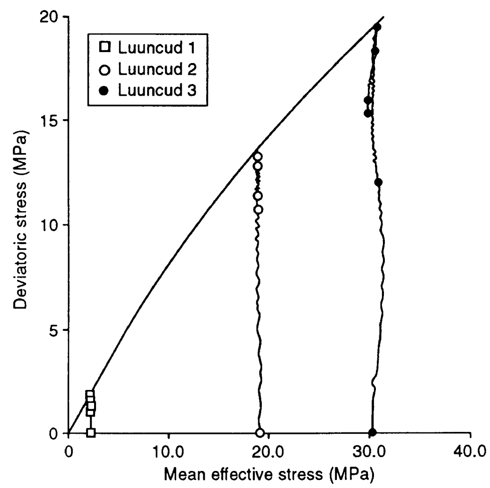

Figure 7. Plot of deviatoric stress against mean effective stress for the Luuncud samples, illustrating the non-linear failure envelope

deformations. Thus although the experiments may not be used to directly predict behavioural changes, such as the strain at which brittle failure will occur, they may be used to infer the mechanisms that cause deformation. The development of microcracks and other structural phenomena within the samples was examined using scanning electron microscopy (SEM). The results of the SEM work are presented elsewhere (Petley, 1994) and are not subject to detailed analysis and interpretation here.

RESULTS

The characteristics of the experiments undertaken in the study are described in Table II. Two sets of deformation experiments were undertaken. The first set (Luuncud1 to Luuncud4) was used to describe the mechanisms of deformation for London Clay throughout the 2–30 MPa stress range. The experiments allowed the form of the failure envelope for London Clay (Figure 7) to be determined. The experiments confirm the conclusion of Bishop *et al*. (1965) that the failure envelope is not linear.

Stress–strain curves for the experiments (Figure 8) allow an analysis of the sample deformation mechanisms. The lower pressure experiments (Luuncud1 and Luuncud2) show a clear peak strength followed by strain-weakening. Examination of the samples after testing showed that in each case a shear plane had developed. Thus the experiments show that at low stresses the London Clay behaves in a brittle manner. The geomorphological implication is that a shallow landslide in unweathered London Clay will undergo sudden and rapid displacement owing to the development of a shear surface. Such behaviour is indeed displayed by natural, shallow landslides in London Clay (Taylor and Spears, 1982) and is similar to that demonstrated in previous experiments (e.g. Skempton and Petley, 1967).

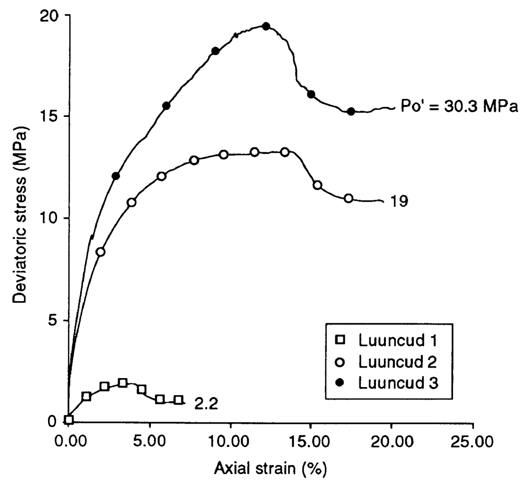

Figure 8. Stress–strain curves for the Luuncud samples

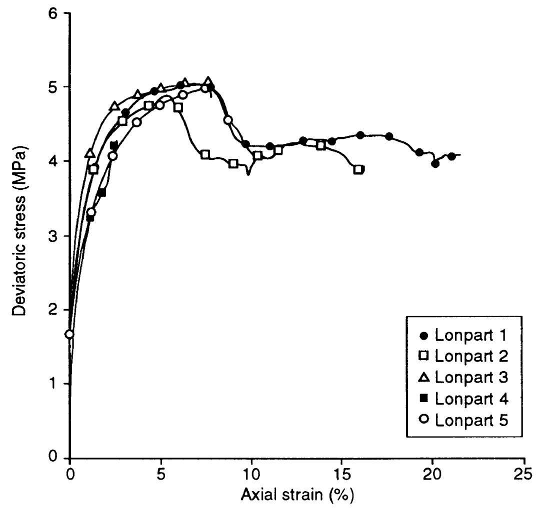

Figure 9. Stress–strain curves for the Lonpart samples

At higher stresses (experiments Luuncud3 and Luuncud4) the behaviour of London Clay changes. The stress–strain curves (Figure 8) show that the samples still undergo brittle failure and strain-weakening to a residual strength. However, behaviour immediately prior to strain-weakening is noticeably different from that of the lower pressure samples. At peak strength the samples do not undergo weakening but retain a constant strength for a considerable further accumulation of strain. Such maintenance of peak strength is usually taken to indicate deformation in the ductile regime. However, as suggested previously, truly ductile deformation can continue to infinite strains, whereas these samples display distinct strain-weakening.

The second phase of the experimental programme was designed to investigate the nature of the anomalous ductile–brittle deformation. This appears to be important in explaining associations between the mechanical behaviour of not only London Clay but soft sediments in general and the landslide activity which develops within them. Five samples were compacted to an effective stress of 6 MPa before being subjected to undrained shear deformation. Each experiment was terminated at a different axial strain, in order to elucidate the mechanisms inducing the behaviour of the samples (Figure 9). The visual examination of samples where tests were terminated during the ductile phase of deformation show that they had not developed a distinct shear plane. However, deformation was not purely ductile but involved the growth of a multitude of pervasive

Figure 10. Scanning electron micrograph of microcrack development

microcracks throughout the sample (Figure 10). Strain accumulated within the sample owing to the growth and coalescence of the microcracks, implying that in this phase the sample was deforming in a uniform, ductile manner but with a mechanism that is brittle on the microscale. Continuing deformation steadily allows the growth and coalescence of microcracks. The coalescing microcracks eventually lead to the formation of a single fracture across the sample. At this point, brittle failure has effectively occurred as the strength of the sample is no longer determined by the intact material. Consequently, strain-weakening is initiated and the sample changes to a brittle style of deformation. This is an important aspect of material behaviour, representing a change in state previously unrecorded, and has some significant implications for the mechanisms of landslide development and the rates of mass movement. These ductile–brittle deformation experiments suggest that as the confining pressure increases, the amount of strain that the sample can accumulate in the microcracking phase, before brittle failure is initiated, will also rise.

DISCUSSION

The results reported here suggest that, whilst at low pressures London Clay behaves in a brittle manner, at elevated pressures it displays a ductile–brittle response. The behaviour at high pressure can be summarized as follows (Figure 11).

I. There is an initial elastic response to loading and at this point all strains are recoverable.
II. At the initiation of yield, interparticle bonding begins to break down. The sample starts to accumulate plastic or irrecoverable strain in addition to elastic strain.
III. As deformation continues, the sample enters a phase of plastic or purely irrecoverable deformation. The main deformation mechanism is the growth of a multitude of pervasive microcracks. Deformation is effectively in a stable state at constant deviatoric stress. Pore pressures in the sample are constant, suggesting that any dilation that is occurring in the sample during the growth of microcracks is being balanced by compaction elsewhere.
IV. The continuing growth and coalescence of the microcracks eventually leads to the formation of a continuous fracture or shear surface across the sample. At this point brittle failure has effectively occurred. The strength of the sample is no longer determined by the properties of the intact material but

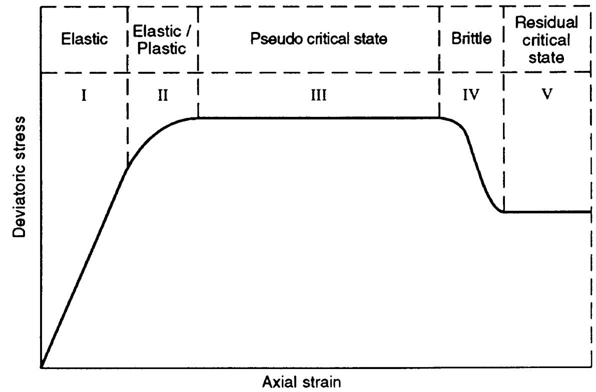

Figure 11. An idealised stress–strain curve illustrating the characteristics of deformation in the transitional regime.

by the friction across the shear plane. As the shear plane develops and becomes smoothed, the sample undergoes strain-weakening.

V. Strain-weakening ends with the development of a complete, smooth, polished shear plane and associated damage zone (Petley *et al*., 1994). The sample is now at a stable state in which deformation can continue without any further change in either the stress regime or strain in the sample.

The deformation of London Clay in the 2–40 MPa range thus illustrates a five-phase pattern of behaviour which is a combination of brittle and ductile deformation. The style of behaviour has been identified in materials as diverse as limestone (Donath *et al*., 1971) and concrete (Chen, 1985), where the behaviour is regarded as the transitional phase between purely brittle and purely ductile deformation. However, this appears to be the first occasion where a transitional phase has been identified for mudrocks.

As described previously, some deep-seated landslides display patterns of movement that are difficult to explain using conventional models in soil mechanics. In particular, some deep-seated landslides display long phases of more or less steady-state creep. The deformation experiments on the London Clay reported here demonstrate that argillaceous rocks show a transitional phase of behaviour. More importantly in geomorphology, the deformation mechanism revealed from the laboratory programme provides a much more comprehensive explanation of the behaviour of deep-seated landslides. The creep phase suggests that mudrocks are able to undergo a phase of deformation in which strains accumulate at constant deviatoric stress. During such a phase the sample is effectively undergoing creep, during which deformation is occurring at an approximately constant rate. The deformation does not require steadily increasing applied stresses but may occur in a static stress state, although small fluctuations in the pore fluid pressure may cause changes in the rate of strain. An increase in pore pressure will not significantly change the deviatoric stress but will alter the effective normal stress. Thus increases in pore pressure may induce increases in the rate of strain, accounting for small changes in the speed of movement.

If the material at the base of a deep-seated landslide is deforming in the transitional regime, a particular pattern of behaviour will be displayed. Whilst the material is in the ductile phase of deformation, the landslide will display creep-like movement. At the base of the landslide the mechanism of deformation or strain will be manifested as the growth of microcracks which eventually coalesce to form a shear surface. At the point at which a shear surface evolves, the material will start to behave in a brittle manner and the friction across the shear surface at this point will be less than the strength of the intact material. As the resistance to shear stress decreases so the rate of strain increases, corresponding to rapidly increasing rates of landslide movement. Landslides that show continuous creep with no sudden failures are representative of a situation where the basal materials are behaving in a truly ductile manner.

The results have two important implications. First, sudden failures of deep-seated landslides in which the materials are deforming in the transitional regime can occur at a fixed strain. When this strain is reached, brittle failure will occur and the rapid strain-weakening will lead to potentially catastrophic rates of movement. Prior to brittle failure, deformation may be evenly distributed throughout a large zone. After failure, deformation will

be concentrated in a distinct shear surface. Second, if the material is deforming in the transitional regime, catastrophic failure is the result of the accumulation of strain and not of changes in the stress state of the landslide system. Assuming that the landslide is moving at an approximately constant rate of creep, the threshold for brittle failure could potentially be reached at a constant deviatoric and effective normal stress. This sudden failure in deep-seated landslides does not necessarily require changes in the pore pressure or changes in the loading of the landslide system. Sudden failure can occur from a steady-state condition if the threshold strain is reached.

CONCLUSION

Although deep-seated landslides have long been identified in the field, their study has largely been restricted to the analysis of morphometry and description of movement rates. Elucidating their mechanical behaviour has been difficult because of the depth of the basal shear surface and associated problems with accessing the zone of deformation. The laboratory study reported here has simulated the stress–strain environment at the bottom of these types of particularly large mass movement. The results suggest that the behaviour of deep-seated landslides is a consequence of the three deformation modes. At the lowest stresses the materials behave in a brittle manner, leading to catastrophic movement as a result of the development of a shear surface. At depth, some materials behave in a purely ductile manner, leading to continual, slow, creep-like movement. The experimental data suggest that some materials may also show a combination of ductile and brittle deformation inducing long periods of creep followed by sudden failure. This is a newly identified phenomenon for mudrocks which goes far in explaining the activity patterns of large, deep-seated terrestrial mass movements. Further research is now required to obtain field data as corroboration of the three modes of deformation, although it is recognized that this will be particularly difficult – if not impossible – to obtain.

ACKNOWLEDGEMENTS

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